VI-5-64 Fundamentals of Design

- Structure trunk stability. Stability formulae for front slope armor on structure trunks are presented in the following tables outlined as follows:

<table>
<thead>
<tr>
<th>Armor Unit</th>
<th>Non-Overtopped</th>
<th>Overtopped</th>
<th>Submerged</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rock</td>
<td>Tables VI-5-22/23</td>
<td>Tables VI-5-24/26</td>
<td>Tables VI-5-25/26</td>
</tr>
<tr>
<td>Concrete cubes</td>
<td>Table VI-5-29</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tetrapods</td>
<td>Table VI-5-30</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Dolosse</td>
<td>Table VI-5-31</td>
<td></td>
<td></td>
</tr>
<tr>
<td>ACCROPODES®</td>
<td>Tables VI-5-32/33</td>
<td></td>
<td></td>
</tr>
<tr>
<td>CORE-LOC®</td>
<td>Table VI-5-34</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tribars</td>
<td>Table VI-5-36</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

- Information on rear side armor stability is given in Table VI-5-28. A formula for stability of reef breakwater is presented in Table VI-5-34. A formula for stability of armor in front of a vertical wall is presented in Table VI-5-35. Rubble-mound structure head stability is given in Tables VI-5-37/38. Parapet walls are placed on top of rubble-mound structures to reduce overtopping by deflecting the uprushing waves back into the sea. This generally reduces the front slope armor stability. A low wall behind a wide front armor berm will hardly affect the armor stability (see Figure VI-5-36a). On the other hand a high wall with a relatively deep foundation situated behind a narrow front armor berm will significantly reduce the armor stability (see Figure VI-5-36b).

![Figure VI-5-36. Illustration of superstructure designs causing insignificant and significant reduction in front slope armor stability](image)

- No generally applicable formulae are available for reduction in front slope armor stability caused by parapet walls.

- Laboratory test limitations. All of the various armor stability criteria represented by the equations and empirical coefficients in Tables VI-5-22 to VI-5-36 were developed in laboratory physical models, most often at reduced scale. Although field experience has added validation to some of these stability formulae, designers should be aware of the following limitations when applying laboratory stability results to prototype conditions.
Table VI-5-22
Rock, Two-Layer Armored Non-Overtopped Slopes (Hudson 1974)

Irregular, head-on waves

\[
\frac{H}{\Delta D_{n,50}} = (K_D \cot \alpha)^{1/3} \quad \text{or} \quad M_{50} = \frac{\rho_v H^3}{K_D (\frac{\rho_v}{\rho_w} - 1)^{3/2} \cot \alpha}
\]  

(VI-5-67)

where

- \( H \) Characteristic wave height (\( H_s \) or \( H_{1/10} \))
- \( D_{n,50} \) Equivalent cube length of median rock
- \( M_{50} \) Medium mass of rocks, \( M_{50} = \rho_v D_{n,50}^3 \)
- \( \rho_v \) Mass density of rocks
- \( \rho_w \) Mass density of water
- \( \Delta \) \((\rho_v/\rho_w) - 1\)
- \( \alpha \) Slope angle
- \( K_D \) Stability coefficient

\( K_D \)-values by SPM 1977, \( H = H_s \), for slope angles \( 1.5 \leq \cot \alpha \leq 3.0 \). (Based entirely on regular wave tests.)

<table>
<thead>
<tr>
<th>Stone shape</th>
<th>Placement</th>
<th>Damage, ( D^1 )</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>0-5%</td>
<td>5-10%</td>
</tr>
<tr>
<td></td>
<td>Breaking waves(^1)</td>
<td>Nonbreaking waves(^2)</td>
</tr>
<tr>
<td>Smooth, rounded</td>
<td>Random</td>
<td>2.1</td>
</tr>
<tr>
<td>Rough angular</td>
<td>Random</td>
<td>3.5</td>
</tr>
<tr>
<td>Rough angular</td>
<td>Special(^3)</td>
<td>4.8</td>
</tr>
</tbody>
</table>

\( K_D \)-values by SPM 1984, \( H = H_{1/10} \).

<table>
<thead>
<tr>
<th>Stone shape</th>
<th>Placement</th>
<th>Damage, ( D^4 = 0-5% )</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Breaking waves(^1)</td>
<td>Nonbreaking waves(^2)</td>
</tr>
<tr>
<td>Smooth rounded</td>
<td>Random</td>
<td>1.2</td>
</tr>
<tr>
<td>Rough angular</td>
<td>Random</td>
<td>2.0</td>
</tr>
<tr>
<td>Rough angular</td>
<td>Special(^3)</td>
<td>5.8</td>
</tr>
</tbody>
</table>

\(^1\) Breaking waves means depth-limited waves, i.e., wave breaking takes place in front of the armor slope. (Critical case for shallow-water structures.)

\(^2\) No depth-limited wave breaking takes place in front of the armor slope.

\(^3\) Special placement with long axis of stone placed perpendicular to the slope face.

\(^4\) \( D \) is defined according to SPM 1984 as follows: The percent damage is based on the volume of armor units displaced from the breakwater zone of active armor unit removal for a specific wave height. This zone extends from the middle of the breakwater crest down the seaward face to a depth equivalent to the wave height causing zero damage below still-water level.


Uncertainty of the formula: The coefficient of variation of Eq VI-5-67 is estimated to be 18% by van der Meer (1988). Melby and Maker (1997) reported a coefficient of variation for \( K_D \) of 25% for stone and 20% for Dolosse.
Table VI-5-23
Rock, Two-Layer Armored Non-Overtopped Slopes (van der Meer 1988)

Irregular, head-on waves

\[
\frac{H_s}{\Delta D_{n50}} = 6.2 \cdot S^{0.2} P^{0.18} N_s^{-0.1} \xi_m^{-0.5} \quad \text{Plunging waves: } \xi_m < \xi_{mc} \quad (VI-5-68)
\]

\[
\frac{H_s}{\Delta D_{n50}} = 1.0 \cdot S^{0.2} P^{-0.13} N_s^{-0.1} (\cot \alpha)^{0.5} \xi_m^p \quad \text{Surging waves: } \xi_m > \xi_{mc} \quad (VI-5-69)
\]

\[
\xi_m = s_m^{-0.5} \tan \alpha \quad \xi_{mc} = \left(6.2 P^{0.31} (\tan \alpha)^{0.5}\right)^{1/(P+0.5)}
\]

where
- \(H_s\) Significant wave height in front of breakwater
- \(D_{n50}\) Equivalent cube length of median rock
- \(\rho_s\) Mass density of rocks
- \(\rho_w\) Mass density of water
- \(\Delta\) \((\rho_s/\rho_w) - 1\)
- \(S\) Relative eroded area (see Table VI-5-21 for nominal values)
- \(P\) Notional permeability (see Figure VI-5-11)
- \(N_s\) Number of waves
- \(\alpha\) Slope angle
- \(s_m\) Wave steepness, \(s_m = H_s/L_{om}\)
- \(L_{om}\) Deepwater wavelength corresponding to mean wave period

Validity:

1) Equations VI-5-68 and VI-5-69 are valid for non-depth-limited waves. For depth-limited waves \(H_s\) is replaced by \(H_{2\%}/1.4\).
2) For \(\cot \alpha \geq 4.0\) only Eq VI-5-68 should be used.
3) \(N_s \leq 7,500\) after which number equilibrium damage is more or less reached.
4) \(0.1 \leq P \leq 0.6\) , \(0.005 \leq s_m \leq 0.06\) , \(2.0 \text{ tonne/m}^3 \leq \rho \leq 3.1 \text{ tonne/m}^3\)
5) For the 8 tests run with depth-limited waves, breaking conditions were limited to spilling breakers which are not as damaging as plunging breakers. Therefore, Eqs VI-5-68 and VI-5-69 may not be conservative in some breaking wave conditions.

Uncertainty of the formula: The coefficient of variation on the factor 6.2 in Eq VI-5-68 and on the factor 1.0 in Eq VI-5-69 are estimated to be 6.5% and 8%, respectively.

Test program: See Table VI-5-4.
Powell and Allsop (1985) analyzed data by Allsop (1983) and proposed the stability formula

\[
\frac{N_{\text{od}}}{N_a} = a \exp \left( b s_p^{-1/3} \frac{H_s}{(\Delta D_{n50})} \right) \quad \text{or} \quad \frac{H_s}{\Delta D_{n50}} = \frac{1}{b} \ln \left( \frac{1}{a} \frac{N_{\text{od}}}{N_a} \right)
\]  

(VI-5-70)

where values of the empirical coefficients \(a\) and \(b\) are given in the table as functions of freeboard \(R_c\) and water depth \(h\). \(N_{\text{od}}\) and \(N_a\) are the number of units displaced out of the armor layer and the total number of armor layer units, respectively.

**Values of coefficients \(a\) and \(b\) in Eqn. VI-5-70.**

<table>
<thead>
<tr>
<th>(R_c/h)</th>
<th>(a \cdot 10^4)</th>
<th>(b)</th>
<th>wave steepness (H_s/L_p)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.29</td>
<td>0.07</td>
<td>1.66</td>
<td>&lt;0.03</td>
</tr>
<tr>
<td>0.39</td>
<td>0.18</td>
<td>1.58</td>
<td>&lt;0.03</td>
</tr>
<tr>
<td>0.57</td>
<td>0.09</td>
<td>1.92</td>
<td>&lt;0.03</td>
</tr>
<tr>
<td>0.38</td>
<td>0.59</td>
<td>1.07</td>
<td>&gt;0.03</td>
</tr>
</tbody>
</table>

van der Meer (1991) suggested that the van der Meer stability formulae for non-overtopped rock slope, Eqns. VI-5-68 and VI-5-69, be used with \(D_{n50}\) replaced by \(f_i D_{n50}\). The reduction factor \(f_i\) is given as

\[
f_i = \left( 1.25 - 4.8 \frac{R_c}{H_s} \sqrt{\frac{s_{op}}{2\pi}} \right)^{-1}
\]  

(VI-5-71)

where \(R_c\) is the freeboard, \(s_{op} = H_s/L_{op}\), and \(L_{op}\) is deep water wave length corresponding to the peak wave period. Limits of Eqn. VI-5-71 are given by

\[
0 < \frac{R_c}{H_s} \sqrt{\frac{s_{op}}{2\pi}} < 0.052
\]

- Some of the earlier results were obtained using monochromatic waves, whereas most of the more recent model tests used irregular waves. Numerous studies have suggested that the monochromatic wave height leading to armor instability roughly corresponds to the significant wave height of irregular waves; however, not all studies have found this correspondence. For preliminary design for nonbreaking wave conditions always use a stability formula based on irregular wave testing if possible. For breaking wave conditions monochromatic wave stability results will be conservative.

- It is generally thought that the higher waves associated with wave groups are responsible for armor layer damage. Typically irregular wave stability model tests use wave trains with assumed random phasing of the spectral components. Over the course of the testing wave groups of differing characteristics impact the structure, and the assumption is that these wave groups are representative of nature. However, it is possible that nonrandom phasing occurs in nature, particularly in shallow water (Andrews and Borgman 1981). Therefore, use of regular wave stability results will be appropriate in some cases.
Table VI-5-25
Rock, Submerged Breakwaters with Two-Layer Armor on Front, Crest and Rear Slope (van der Meer 1991)

Irregular, head-on waves

\[
\frac{h_c'}{h} = (2.1 + 0.1 \ S) \ \exp(-0.14 \ N_s^*)
\]  

(VI-5-72)

where  

- \( h \)  Water depth
- \( h_c' \)  Height of structure over seafloor level \((h - h_c'\) is the water depth over the structure crest).
- \( S \)  Relative eroded area
- \( N_s^* \)  Spectral stability number, \( N_s^* = \frac{H_s}{\Delta D_{95/0}} s_p^{1/3} \)

Uncertainty of the formula: The uncertainty of Eq VI-5-72 can be expressed by considering the factor 2.1 as a Gaussian distributed stochastic variable with mean of 2.1 and standard deviation of 0.35, i.e., a coefficient of variation of 17%.

Data source: Givler and Sorensen (1986): regular head-on waves, slope 1:1.5
van der Meer (1991): irregular head-on waves, slope 1:2
Table VI-5-26
Rock, Two-Layer Armored Low-Crested and Submerged Breakwaters (Vidal et al. 1992)

Tested trunk cross section

Tested ranges
Irregular, head-on waves
Spectral $H_s=5-19$ cm, $T_p=1.4$ and 1.8 sec.
Free board: $-5cm \leq R_c = h_c - h \leq 6cm$
Dimensionless freeboard: $-2 \leq R_c / D_{n50} \leq 2.4$

Stability corresponding to initiation of damage, $S=0.5-1.5$

Stability corresponding to extraction of some rocks from lower layer, $S=2.0-2.5$. 
Table VI-5-27
Rock, Low-Crested Reef Breakwaters Built Using Only One Class of Stone

Irregular, head-on waves

van der Meer (1990)

Trunk cross section of reef breakwater

The equilibrium height of the structure

\[ h_c = \sqrt{\frac{A_t}{\exp(a \cdot N_s^*)}} \text{ with a maximum of } h'_c \]  \hspace{1cm} (VI–5–73)

where  
\[ A_t \]  area of initial cross section of structure
\[ h \]  water depth at toe of structure
\[ h'_c \]  initial height of structure
\[ N_s^* = \frac{h_s}{\Delta D_{S50}} \delta_p^{1/3} \]
\[ a = -0.028 + 0.045 \frac{A_t}{(h_s)^2} + 0.034 \frac{h'_c}{h} - 6 \times 10^{-9} \frac{A_t^2}{D_{S50}^2} \]

Data source: Ahrens (1987), van der Meer (1990)

Powell and Alsop (1985) analyzed data by Ahrens, Viggossen, and Zirkle (1982) and Ahrens (1984) and proposed the stability formula

\[ \frac{N_{ol}}{N_a} = a \exp \left[ b \frac{H_s}{(\Delta D_{S50})} \right] \quad \text{or} \quad \frac{H_s}{\Delta D_{S50}} = \frac{1}{b} \ln \left( \frac{1}{ \frac{N_{ol}}{N_a} } \right) \]  \hspace{1cm} (VI–5–74)

where values of the empirical coefficients \( a \) and \( b \) are given in the table as functions of freeboard \( R_c \) and water depth \( h \). \( N_{ol} \) and \( N_a \) are the number of displaced rocks and the total number of rocks in the mound, respectively.

Values of coefficients \( a \) and \( b \) in Eq (VI-5-74).

<table>
<thead>
<tr>
<th>( \frac{R_c}{h} )</th>
<th>( a \cdot 10^4 )</th>
<th>( b )</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0</td>
<td>15</td>
<td>0.31</td>
</tr>
<tr>
<td>0.2</td>
<td>17</td>
<td>0.33</td>
</tr>
<tr>
<td>0.4</td>
<td>4.8</td>
<td>0.53</td>
</tr>
</tbody>
</table>

Valid for \( 0.0012 < \frac{H_s}{L_p} < 0.036 \)
Irregular, head-on waves

Jensen (1984) reported results from two case studies of conventional rock armored rubble-mound breakwaters with the main armor carried over the crests and the upper part of the rear slope. Crest width was approximately 3–4 stone diameters. Although Jensen points out that the results are very project dependent, these results could be useful for preliminary estimates. Wave steepness significantly influences the rear side damage.

\[
\frac{Crest \ level - WL}{H_s}
\]

\[
TP = 20.1 \text{ s (Midvaag), } s = 0.015
\]

\[
TP = 14.1 \text{ s (Midvaag), } s = 0.021
\]

\[
TP = 7.85 \text{ s (Korser), } s = 0.036
\]

- Hand-built armor layers on laboratory structures could be tighter than are armor layers typically constructed in the prototype. This leads to unconservative stability results. In particular special placement of armor in the laboratory is unlikely to be reproduced as well on the job site, especially below the water surface where placement will be much more random. For this reason it may be advisable to use stability criteria for random placement as a basis for design.

- Armor stability formulae are intended for use in preliminary design phases and for estimating material quantities. When feasible, preliminary designs should be confirmed and optimized with hydraulic model tests.
van der Meer (1988b)

\[ N_s = \frac{H_s}{\Delta D_n} = \left( 6.7 \frac{N_{od}^{0.4}}{N_z^{0.3} + 1.0} \right) s_m^{-0.1} \]  \hspace{1cm} (VI-5-75)

where \( H_s \) Significant wave height in front of breakwater  
\( \rho_s \) Mass density of concrete  
\( \rho_w \) Mass density of water  
\( \Delta \) \( \left( \frac{\rho_s}{\rho_w} \right) - 1 \)  
\( D_n \) Cube length  
\( N_{od} \) Number of units displaced out of the armor layer within a strip width of one cube length \( D_n \)  
\( N_z \) Number of waves  
\( s_m \) Wave steepness, \( s_m = H_s/L_{am} \)

Valid for: Non-depth-limited wave conditions. Irregular head-on waves  
Two layer cubes randomly placed on 1:1.5 slope  
Surf similarity parameter range \( 3 < \xi_m < 6 \)

Uncertainty of the formula: corresponds to a coefficient of variation of approximately 0.10

Brosen, Burchart, and Larsen (1974)

Brosen, Burchart, and Larsen gave the following average \( N_s \) and corresponding \( K_D \)-values for a two layer concrete cube armor, random placement, slope angles \( 1.5 \leq \cot \alpha \leq 2.0 \) and non-depth-limited irregular waves

<table>
<thead>
<tr>
<th>Damage level</th>
<th>( N_s = \frac{H_s}{\Delta D_n} )</th>
<th>( K_D )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Onset, ( D = 0% )</td>
<td>1.8 - 2.0</td>
<td>3.9 - 5.3</td>
</tr>
<tr>
<td>Moderate, ( D = 4% )</td>
<td>2.3 - 2.6</td>
<td>8.1 - 12</td>
</tr>
</tbody>
</table>
van der Meer (1988b) for non-depth-limited waves

\[ N_s = \frac{H_s}{\Delta D_n} = \left( 3.75 \frac{N_{od}^{0.5}}{N_{z}^{0.25}} + 0.85 \right) s_{om}^{-0.2} \]  

(VI-5-76)

where \( H_s \)  
Significant wave height in front of breakwater

\( \rho_c \)  
Mass density of concrete

\( \rho_w \)  
Mass density of water

\( \Delta \)  
\( (\rho_c/\rho_w) - 1 \)

\( D_n \)  
Equivalent cube length, i.e., length of cube with the same volume as Tetrapods

\( N_{od} \)  
Number of units displaced out of the armor layer within a strip width of one cube length \( D_n \)

\( N_z \)  
Number of waves

\( s_{om} \)  
Wave steepness, \( s_{om} = H_s / L_{om} \)

Valid for: Non-depth limited wave conditions, Irregular head-on waves

Two layer tetrapods on 1:1.5 slope

Surf similarity parameter range 3.5 < \( \xi_m \) < 6

Uncertainty of the formula: corresponds to a coefficient of variation of approximately 0.10

d’Angremond, van der Meer, and van Nes (1994) for depth-limited waves

\[ N_s = \frac{H_{2\%}}{\Delta D_n} = 1.4 \left( 3.75 \frac{N_{od}^{0.5}}{N_{z}^{0.25}} + 0.85 \right) s_{om}^{-0.2} \]  

(VI-5-77)

In deep water the ratio \( H_{2\%} / H_s = 1.4 \) for Rayleigh distributed waves. In shallow water this ratio decreases with decreasing relative water depth due to wave breaking.
Table VI-5-31
Dolos, Non-Overtopped Slopes (Burcharth and Liu 1992)

\[ N_s = \frac{H_s}{\Delta D_n} = (47 - 72r) \varphi_{n=2}D_{1/3}N_{z}^{-0.1} \]

\[ = (17 - 26r) \varphi_{n=2}^{2/3}N_{od}^{1/3}N_{z}^{-0.1} \quad (VI-5-78) \]

where \( H_s \) Significant wave height in front of breakwater
\( \rho_s \) Mass density of concrete
\( \rho_w \) Mass density of water
\( \Delta \) \((\rho_s/\rho_w) - 1\)
\( D_n \) Equivalent cube length, i.e., length of cube with the same volume as dolos
\( r \) Dolos waist ratio
\( \varphi \) Packing density
\( D \) Relative number of units within levels SWL ± 6.5 \( D_n \) displaced one dolos height \( h \), or more (e.g., for 2% displacement insert \( D = 0.02 \))
\( N_{od} \) Number of displaced units within a strip width of one equivalent cube length \( D_n \)
\( N_z \) Number of waves. For \( N_z \geq 3000 \) use \( N_z = 3000 \)

Valid for: Breaking and nonbreaking wave conditions
Irregular head-on waves, two layer randomly placed dolos with a 1:1.5 slope
\[ 0.32 < r < 0.42 \]
\[ 0.61 < \varphi < 1 \]
\[ 1\% < D < 15\% \]
\[ 2.49 < \xi_0 < 11.7 \]

Slope angle: The effect of the slope angle on the hydraulic stability is not included in Eq VI-5-78. Brorsen, Burcharth, and Larsen (1974) found only a marginal influence for slopes in the range 1:1 to 1:3.

Uncertainty of the formula: corresponds to a coefficient of variation of approximately 0.22.

(continued)
Fit of hydraulic stability formula for a two-layer randomly placed dolos armor on a slope of 1:1.5. Damage levels, $D = 2\%$, $5\%$ and $10\%$ displaced units within levels SWL $\pm 6.5D_n$. 

<table>
<thead>
<tr>
<th>Reference</th>
<th>$\phi_{n-2}$</th>
<th>Repeated No</th>
<th>Duration (min.)</th>
<th>$\xi_{mo}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bronsøe et al. (1974)</td>
<td>1 (App.)</td>
<td>2</td>
<td>60</td>
<td>2.49-5.37</td>
</tr>
<tr>
<td>Burcharth et al. (1988)</td>
<td>0.61-0.7</td>
<td>5 or 15</td>
<td>20</td>
<td>3.04-4.49</td>
</tr>
<tr>
<td>Hofmann et al. (1990)</td>
<td>1</td>
<td>3 or 8</td>
<td>60</td>
<td>2.81-7.6</td>
</tr>
<tr>
<td>Burchardt et al. (1992)</td>
<td>0.74</td>
<td>20</td>
<td>5</td>
<td>3.23-11.7</td>
</tr>
</tbody>
</table>
Table VI-5-32
ACCROPODE © (van der Meer 1988b)

\[ N_s = \frac{H_s}{\Delta D_n} = \begin{cases} 
3.7 & \text{no damage} \\
4.1 & \text{failure} 
\end{cases} \quad \text{(VI-5-79)} \]

where \( H_s \) Significant wave height in front of breakwater
\( \rho_s \) Mass density of concrete
\( \rho_w \) Mass density of water
\( \Delta \) \( (\rho_s/\rho_w) - 1 \)
\( D_n \) Equivalent cube length, i.e., length of cube with the same volume as Accropode

Valid for: Irregular, head-on waves
Nonbreaking wave conditions
One layer of Accropodes on slope 1:1.33 placed in accordance with
SOGREAH recommendations
No influence of number of waves were found except after start of failure.

Uncertainty of the formula: The standard deviation of the factors 3.7 and 4.1 is approximately 0.2.
Table VI-5-33
ACCROPODE ®, Non-Overtopped or Marginally Overtopped Slopes (Burcharth et al. 1998)

\[ N_s = \frac{H_s}{\Delta D_n} = A \left( D^{0.2} + 7.70 \right) \text{ or } D = 50 \left( \frac{H_s}{\Delta D_n} - 3.54 \right)^5 \]  

where \( H_s \): Significant wave height in front of breakwater
\( \rho_o \): Mass density of concrete
\( \rho_w \): Mass density of water
\( \Delta \): \( \left( \frac{\rho_o}{\rho_w} \right) - 1 \)
\( D_n \): Equivalent cube length, i.e., length of cube with the same volume as Accropode
\( D \): Relative number of units displaced more than distance \( D_n \)
\( A \): Coefficient with mean value \( \mu = 0.46 \) and coefficient of variation \( \sigma/\mu = 0.02 + 0.05(1 - D)^{1/5} \), where \( \sigma \) is the standard deviation

Valid for: Irregular, head-on waves
Breaking and nonbreaking wave conditions

One layer of Accropodes on 1:1.33 slope placed in accordance with SOGREAH recommendations
Accropodes placed on filter layer and conventional quarry rock run
\( 3.5 < \xi_m < 4.5 \) (minimum stability range, see figure)
No influence of number of waves were found except after start of failure.

Uncertainty of the formula: see figure and explanation of \( A \).

SOGREAH recommends for preliminary design the following \( K_D \)-values to be used in Eq VI-5-67.

\[ K_D = \begin{cases} 
15 & \text{Nonbreaking waves} \\
12 & \text{Breaking waves} 
\end{cases} \]
Irregular, head-on waves

\[
\frac{H}{\Delta D_{n50}} = (K_D \cot \alpha)^{1/3} \quad \text{or} \quad \frac{M_{50}}{K_D} = \frac{\rho_c H^3}{(\rho_c/\rho_w - 1)^3 \cot \alpha}
\]

where
\( H \) Characteristic wave height (\( H_s \))
\( D_{n50} \) Equivalent length of cube having same mass as Core-Loc, \( D_{50} = (M_{50}/\rho_c)^{1/3} \)
\( M_{50} \) Mass of Core-Loc armor unit, \( M_{50} = \rho_c (D_{n50})^3 \)
\( \rho_c \) Mass density of concrete
\( \rho_w \) Mass density of water
\( \Delta \) \( (\rho_c/\rho_w) - 1 \)
\( \alpha \) Slope angle
\( K_D \) Stability coefficient

**Trunk section stability.** Melby and Turk (1994) found no reasonable \( (K_D < 50) \) irregular breaking or nonbreaking wave conditions that would destabilize the layer. For an armor layer exposed to regular depth-limited plunging to collapsing waves, \( K_D = 16 \) in Equation VI-5-81 is recommended for preliminary design of all trunk sections. The recommended value of \( K_D \) is conservative, and it represents a zero-damage condition with little to no armor unit rocking. Site specific physical model tests will usually yield higher values.

**Head section stability.** \( K_D = 13 \) is recommended for preliminary design of head sections exposed to both breaking and nonbreaking oblique and head-on waves.

**Stability test parameters**

- **Model parameters**
  \( M_{50} = 219 \text{ g}; \) Depths: 36 and 61 cm; Height: 90 cm
- **Wave parameters**
  \( 4.6 \leq H_{m0} \leq 36 \text{ cm}; \ 1.5 \leq T_p \leq 4.7 \text{ sec} \)
- **Structure slope,} \( \alpha \)
  \( 1V:1.33H \) and \( 1V:1.5H \)
- **Surf similarity parameter**
  \( 2.13 \leq \zeta_o \leq 15.9 \)
- **Relative depth**
  \( 0.012 \leq d/L_o \leq 0.175 \)
- **Wave steepness**
  \( 0.001 \leq H_{m0}/L_o \leq \text{breaking} \)

**Placement.** Core-Locs are intended to be randomly placed in a single-unit thick layer on steep or shallow slopes. They are well suited for use in repairing existing dolos structures because they interlock well with dolosse when properly sized (length of Core-Loc central flume is 92 percent of the dolosse fluke length).
Table VI-5-35
Tetrapods, Horizontally Composite Breakwaters (Hanzawa et al. 1996)

\[ N_s = \frac{H_s}{\Delta D_n} = 2.32 \left( \frac{N_{od}}{N_z^{0.5}} \right)^{0.2} + 1.33 \]  \hspace{1cm} (VI-5-82)

where  
- \( H_s \): Significant wave height in front of breakwater  
- \( \rho_s \): Mass density of concrete  
- \( \rho_w \): Mass density of water  
- \( \Delta \): \( (\rho_s/\rho_w) - 1 \)  
- \( D_n \): Equivalent cube length, i.e., length of cube with the same volume as Tetrapods  
- \( N_{od} \): Number of units displaced out of the armor layer within a strip width of one cube length \( D_n \)  
- \( N_z \): Number of waves

Test range: The formula was obtained by fitting of earlier model test results and five real project model tests  
- Irregular head-on waves  
- Water depth: 0.25 - 0.50 cm  
- Slope: 1:1.5  
- Foreshore: 1:15 - 1:100  
- Mass of Tetrapods: 90 - 700 g  
- \( H_s \): 8 - 25.9 cm; \( T_s \): 1.74 - 2.5 s; \( s_{om} \): 0.013 - 0.04

Uncertainty of the formula: Not given. Tanimoto, Hanashika, and Yamazaki (1985) gave the standard deviation of \( N_{od} \) equal to \( 0.36 N_{od}^{0.5} \)
Regular, head-on waves

\[ \frac{H}{\Delta D_{n50}} = (K_D \cot \alpha)^{1/3} \quad \text{or} \quad M_{50} = \frac{\rho_s H^3}{K_D \left( \frac{\rho_s}{\rho_w} - 1 \right)^3 \cot \alpha} \]  

(VI-5-83)

where

- \( H \) Characteristic wave height (\( H_s \))
- \( D_{n50} \) Equivalent cube length of median rock
- \( M_{50} \) Median mass of stone armor unit, \( M_{50} = \rho_s (D_{n50})^3 \)
- \( \rho_s \) Mass density of stone
- \( \rho_w \) Mass density of water
- \( \Delta \) \( (\rho_s / \rho_w) - 1 \)
- \( \alpha \) Slope angle
- \( K_D \) Stability coefficient

Trunk section stability.

\( K_D \)-values by Shore Protection Manual, \( H = H_{1/10}, 0\% \) to 5\% damage

<table>
<thead>
<tr>
<th>Placement</th>
<th>Layers</th>
<th>Breaking waves(^1)</th>
<th>Nonbreaking waves(^2)</th>
<th>Slope angle (\cot \alpha)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Random</td>
<td>2</td>
<td>9.0</td>
<td>10.0</td>
<td>1.5 - 3.0</td>
</tr>
<tr>
<td>Pattern-placed</td>
<td>1</td>
<td>12.0</td>
<td>15.0</td>
<td>(not given)</td>
</tr>
</tbody>
</table>

\(^1\) Depth-limited breaking with waves breaking in front of and on the armor slope.

\(^2\) No depth-limited breaking occurs in front of the armor slope.

Design wave height considerations. In shallow water the most severe wave condition for design of any part of a rubble-mound structure is usually the combination of predicted water depth and extreme incident wave height and period that produces waves which would break directly on the structure. In some cases, particularly for steep foreshore slopes, waves breaking offshore will strike directly on the structure. Goda (1985) recommended computing the design wave height a distance 5\( H_s \) from the structure toe to account for the travel distance of large breakers. A shallow-water coastal structure exposed to a variety of water depths, especially a shore-perpendicular structure such as a groin, should have wave conditions investigated for each range of water depths to determine the highest breaking wave that might impact any part of the structure. For example, a groin that normally experiences wave forces on its armor layer near the seaward end might become submerged during storm surges, and the worst breaking wave condition could occur on a more...
landward portion of the groin. The effect of oblique wave approach on armor layer stability has not yet been sufficiently quantified. Tests in the European Marine Science and Technology (MAST) program seemed to indicate relatively little reduction in damage for rock armored slopes subjected to oblique wave approach angles up to 60 deg compared to waves of normal incidence (Allsop 1995). The stability of any rubble-mound structure exposed to oblique wave attack should be confirmed with physical model tests.

(6) Structure head section stability.

(a) Under similar wave conditions the round head of a rubble-mound structure normally sustains more extensive and more frequent damage than the structure trunk. One reason is very high cone-overflow velocities, sometimes enhanced in certain areas by wave refraction. Another reason is the reduced support from neighboring units in the direction of wave overflow on the lee side of the cone as shown in Figure VI-5-37. This figure also illustrates the position of the most critical area for armor layer instability. The toe within the same area is also vulnerable to damage in shallow-water situations, and a toe failure will often trigger failure of the armor layer see Part VI-5-6-b-2, “Scour at sloping structures.”

(b) Table VI-5-37 presents stability criteria for stone and dolos rubble-mound structure heads subjected to breaking and nonbreaking waves without overtopping, and Table VI-5-38 gives stability criteria for tetrapod and tribar concrete armor units.

![Figure VI-5-37. Illustration of critical areas for damage to armor layers in the round head (Burcharth 1993)]
Rock and dolos armor, monochromatic waves
Mostly monochromatic waves with a few irregular wave cases
Breaking and nonbreaking waves
Incident wave angles: 0°, 45°, 90°, 135° (note: 0° is wave crests perpendicular to trunk)

\[
\frac{H}{\Delta D_{n50}} = A \xi^2 + B \xi + C_c
\]  

\(\xi = \frac{\tan \alpha}{(H/L)^{1/2}}\)

where

- \(H\): Characteristic wave height
- \(D_{n50}\): Equivalent cube length of median rock
- \(\rho_s\): Mass density of stone
- \(\rho_w\): Mass density of water
- \(\Delta\): \((\rho_s/\rho_w) - 1\)
- \(L\): Local wavelength at structure toe
- \(\alpha\): Structure armor slope
- \(A, B, C_c\): Empirical coefficients

Table of coefficients for use in Equation VI-5-84

<table>
<thead>
<tr>
<th>Armor Type</th>
<th>A</th>
<th>B</th>
<th>C_c</th>
<th>Slope</th>
<th>Range of (\xi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stone</td>
<td>0.272</td>
<td>-1.749</td>
<td>4.179</td>
<td>1V to 1.5H</td>
<td>2.1 - 4.1</td>
</tr>
<tr>
<td>Stone</td>
<td>0.198</td>
<td>-1.234</td>
<td>3.289</td>
<td>1V to 2.0H</td>
<td>1.8 - 3.4</td>
</tr>
<tr>
<td>Dolos</td>
<td>0.406</td>
<td>-2.800</td>
<td>6.881</td>
<td>1V to 1.5H</td>
<td>2.2 - 4.4</td>
</tr>
<tr>
<td>Dolos</td>
<td>0.840</td>
<td>-4.466</td>
<td>8.244</td>
<td>1V to 2.0H</td>
<td>1.7 - 3.2</td>
</tr>
</tbody>
</table>

Notes: The curves giving the best fit to the data were lowered by two standard deviations to provide a conservative lower envelope to the stability results.

A limited number of tests using irregular waves produced corresponding results with \(T_p\) equivalent to the monochromatic period and \(H_{m0}\) equal to the monochromatic wave height.
Regular, head-on waves

\[
\frac{H}{\Delta D_{n50}} = (K_D \cot \alpha)^{1/3} \quad \text{or} \quad M_{50} = \frac{\rho_s H^3}{K_D (\frac{\rho_s}{\rho_w} - 1)^3 \cot \alpha}
\]  

(VI-5-85)

where

- \( H \): Characteristic wave height (\(H_s\))
- \( D_{n50} \): Equivalent cube length of median rock
- \( M_{50} \): Median mass of stone armor unit, \(M_{50} = \rho_s (D_{n50})^3\)
- \( \rho_s \): Mass density of stone
- \( \rho_w \): Mass density of water
- \( \Delta \): \((\rho_s/\rho_w) - 1\)
- \( \alpha \): Slope angle
- \( K_D \): Stability coefficient

Head Section Stability.

\(K_D\)-values by Shore Protection Manual (1984), \(H = H_{1/10}\), 0 percent to 5 percent damage

<table>
<thead>
<tr>
<th>Armor Unit</th>
<th>Placement</th>
<th>Layers</th>
<th>Breaking Waves(^1)</th>
<th>Nonbreaking Waves(^2)</th>
<th>Slope Angle (\text{cot} \alpha)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tetrapod</td>
<td>Random</td>
<td>2</td>
<td>5.0(^1)</td>
<td>6.0</td>
<td>1.5</td>
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<td></td>
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<td></td>
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<td>3.5</td>
<td>4.0</td>
<td>3.0</td>
</tr>
<tr>
<td>Tribar</td>
<td>Random</td>
<td>2</td>
<td>8.3</td>
<td>9.0</td>
<td>1.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>7.8</td>
<td>8.5</td>
<td>2.0</td>
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<td></td>
<td></td>
<td>6.0</td>
<td>6.5</td>
<td>3.0</td>
</tr>
<tr>
<td>Tribar</td>
<td>Pattern</td>
<td>1</td>
<td>7.5</td>
<td>9.5</td>
<td>(not given)</td>
</tr>
</tbody>
</table>

\(^1\) Depth-limited breaking with waves breaking in front of and on the armor slope.
\(^2\) No depth-limited breaking occurs in front of the armor slope.
\(^3\) \(K_D\) values shown in italics are unsupported by tests results and are provided only for preliminary design purposes.